

COMPARISON OF EXISTING STRENGTH CRITERIA FOR CONFINED MASONRY WALLS

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Resumen. La construcción de estructuras de mampostería encadenada comienza luego del terremoto de Reggio-Messina en 1908 y se convierte en una de las construcciones de vivienda más ampliamente utilizada en el mundo y en particular en Latinoamérica. En este trabajo se presentan los resultados de la comparación de expresiones teóricas, para la verificación de la resistencia de estructuras de mampostería encadenada, de siete normas de construcciones sismorresistentes. Los resultados incluyen la comparación de curvas teóricas mínimas resultantes de la mínima curva obtenida entre la curva de resistencia de corte y la curva de resistencia a flexo-compresión. Los resultados muestran que dichas curvas mínimas corresponden a la curva dada por la resistencia a corte hasta por lo menos un valor de 0.55 de la carga de compresión axial máxima. Las expresiones teóricas son luego comparadas con resultados experimentales de pruebas de laboratorio llevadas a cabo en la actualidad. En general todas las expresiones teóricas de las normas sismorresistentes dan valores aproximados a los obtenidos experimentalmente con excepción de la norma Colombiana y Costa Rica cuyos valores teóricos tienden a ser bastante conservativos.

Palabras clave: Mampostería encadenada, resistencia al corte, resistencia a flexo-compresión, resultados experimentales

Abstract. *The construction of confined masonry started after the 1908 Reggio-Messina earthquake and it has become one of the most common and inexpensive structural construction systems used for housing. The aim of this paper is to present a comparison of the expressions of the strength criteria proposed by different codes and their capability of predicting the experimental response. Seven different formulations for the lateral resistance of confined masonry panels have been considered. The results include the comparison of the minimum shear force-axial compression interaction domains given by the code formulations. Results show that most of the minimum curves are determined by the shear resistance expression at least until a value of 0.55 of vertical load capacity ratio. Theoretical expressions are then compared to experimental results from tests performed worldwide. All codes provide a good approximation of the experimental results with the exception of the Colombian and Costa Rican seismic codes which tend to be very conservative.*

Keywords: *Confined masonry, shear strength, theoretical flexural capacity, experimental results*

INTRODUCTION

Confined masonry has evolved essentially through an informal process based on experience and it has been incorporated in formal construction via code requirements and design procedures. The construction system was proposed in 1909 after the Reggio-Messina earthquake. This event determined the opportunity to develop a new construction technique, the “*confined masonry*”, to rebuilt entire dwellings affected by the earthquake. The structural system was constituted by a reinforced concrete framework connected to the bearing masonry wall. On the other hand, in Latin America, Chile seems to have the longest history related to confined masonry practice dating back to the 1930s. Some confined masonry buildings were reportedly affected by the 1939 Chilean earthquake ($M = 7.8$).

Several formulations for the lateral strength of confined masonry are available in different country codes. In order to predict the resistance given by each code formulation a case study configuration of confined masonry panel was considered. The expressions used correspond to the following codes:

- 1) Argentinean seismic code, INPRES-CIRSOC 103 ¹
- 2) Mexican seismic code for masonry structures, NTCM 2004 ²
- 3) Chilean seismic code, NCh2123 ³
- 4) Colombian seismic code, NSR-98 ⁴
- 5) Costa Rican seismic code, CSCR-02 ⁵
- 6) Peruvian seismic code, E.070 ⁶
- 7) Eurocode 6 ⁷ (masonry structures) and Eurocode 8 (seismic design) ⁸

DESCRIPTION AND EVALUATION OF THEORETICAL EXPRESSIONS

The theoretical resistance of a confined masonry panel was calculated for an arbitrary panel configuration for each code formulation with the corresponding hypotheses (Figure 1). The considered confined masonry panel is a 4 m long, 3 m height and 0.24 m thickness. The longitudinal reinforcement of the vertical confining elements is assumed to be 4 $\varnothing$ 8 mm steel bars with yield strength of 420 MPa. The dimensions of the cross section of the confining elements are 15 cm x 15 cm. The resistance parameters are $f_m = 1.5$ MPa for the mean compressive strength of the masonry and $\tau_m = 0.30$ MPa for the mean shear strength.

The value of the shear strength of the masonry used in the different code formulations results to be different according to the testing procedures and their interpretation. In the Latin American codes this value, called τ_m , is determined testing square masonry wall segments in diagonal compression according to the procedures of the Argentinean, Chilean, Mexican or Peruvian standards ($\tau_m = 0.7P/d.e$; where P is the vertical load, d is the length of the side of the square masonry wall and e is the thickness of the square masonry wall). On the other hand, in the Eurocode the value of initial shear strength, f_{vo} , is determined by a test setup with brick-triplets according to EN 1052-3, where the arrangement gives the advantage that bricks of all shapes can be used with different degrees of prestressing normal to the bed-joints.

From the comparison of expressions of Latin American seismic codes and the Eurocode to obtain the shear strength with the two different procedures mentioned above (Magenes et al. ⁹) the ratio between f_{vo} and τ_m results to be approximately 0.60 ($f_{vo} = 0.6 \tau_m$).

Finally the mean compressive strength of the concrete is 17.5 MPa.

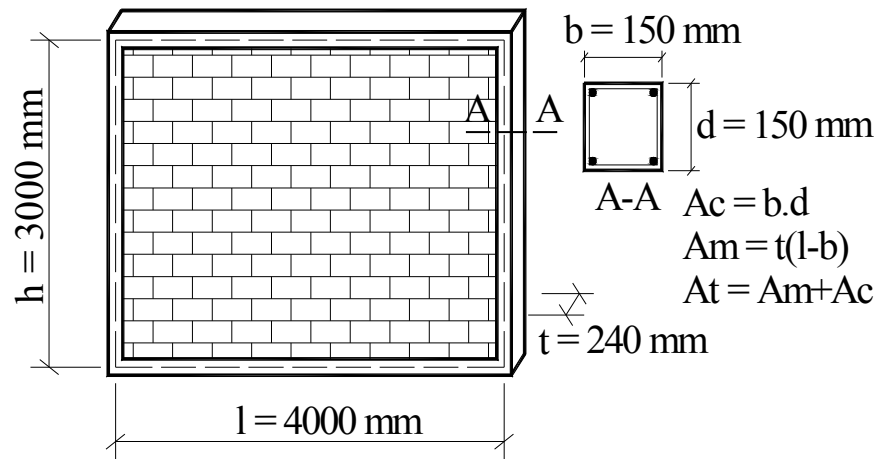


FIGURE 1 Geometrical dimensions of the specimen

Different expressions are used to compute the strength of confined masonry walls (Table 1). To compute the shear strength, V_{res} , the expressions are based on a Mohr-Coulomb type of failure criterion. The shear strength is a function of the initial shear strength of the masonry, a factor which takes into account the effects of seismic load reversal, the axial vertical loads (N), the gross section of the confined masonry or the

masonry panel and a friction coefficient which in most of the cases is assumed approximately equal to 0.3.

On the other hand the theoretical flexural capacity under zero vertical loads (M_o) is a function of the gross section of the longitudinal reinforcement (A_s), the yield strength of steel bars (f_y) and the length between tie-columns axes. When vertical loads are acting they also contribute to the flexural strength.

Assumptions used by codes are listed below from 1 to 7, as the designation given above and in Table 1:

1) In the Argentinean code the shear strength is calculated as a function of the initial shear strength (τ_m) multiplied by a factor equal to 0.6. Only 85 % of the applied vertical loads are considered and a friction coefficient of 0.3 is used. The area corresponds to the gross section of the masonry (A_m).

In the theoretical flexural capacity the distance between column axes is used and vertical loads are consider to increase the strength until a limit given by the maximum compression strength divided by 3. When vertical loads exceed this limit they produce a decrease on the flexural resistance.

TABLE 1 Lateral strength expressions for confined masonry walls

Code	Shear strength expressions	Flexural strength expressions
1	$V_{res} = (0.6\tau_m A_m + 0.255 N) \leq 1.5\tau_m A_m$	$M_{res} = \begin{cases} M_o + 0.3N(l-b) & \text{if } N \leq \frac{P_o}{3} \\ (1.5M_o + 0.15P_o(l-b))(1-\frac{N}{P_o}) & \text{if } N > \frac{P_o}{3} \end{cases}$ Where $P_o = f_m A_m$ and $M_o = A_s f_y l$
2	$V_{res} = (0.5\tau_m A_t + 0.3N) \leq 1.5\tau_m A_t$	$M_{res} = \begin{cases} M = M_o + 0.3N(l+\frac{b}{2}) & \text{if } N \leq \frac{P}{3} \\ M = (1.5M_o + 0.15P(l+\frac{b}{2}))(1-\frac{N}{P}) & \text{if } N > \frac{P}{3} \end{cases}$ Where $M_o = A_s f_y l$
3	$V_{res} = (0.57\tau_m A_t + 0.3N) \leq 0.87\tau_m A_t$	$M_{res} = \begin{cases} M = M_o + 0.2N(l+\frac{b}{2}) & \text{if } N \leq \frac{P}{3} \\ M = (1.5M_o + 0.10P(l+\frac{b}{2}))(1-\frac{N}{P}) & \text{if } N > \frac{P}{3} \end{cases}$ Where $M_o = 0.9 A_s f_y l$
4	$V_{res} = (\frac{\sqrt{f_m}}{12} + \frac{N}{3A_t})A_m \leq \frac{\sqrt{f_m}}{6} A_m$	$M_{res} = A_{se} f_y ((l+\frac{b}{2}) - \frac{a}{2}) \quad \text{if } N \leq 0.10 f_m A_t$ Where $A_{se} = \frac{(A_s f_y + N)}{f_y}$ and $a = \frac{(A_s f_y + N)}{0.85 f_m t}$
5	$V_{res} = (0.50\sqrt{f_m} + 0.30\frac{N}{A_m})0.8lt$ $V_{res} \leq (7.5 + 0.30\frac{N}{A_m})0.8lt$	$M_{res} = A_s f_y \frac{(l+b)}{2} (1 - \frac{N}{A_s f_y}) (1 - \frac{c}{(l+b)})$ Where $\frac{c}{(l+b)} = \frac{(\alpha + \beta)}{(2\alpha + 0.72)}$ and $\alpha = \frac{(A_s f_y)}{(t(l+b)f_c)}$ and $\beta = \frac{N}{(t(l+b)f_c)}$
6	$V_{res} = 0.5\tau_m A_t + 0.23N$	$M_{res} = A_s f_y 0.8(l+b) + \frac{N(l+b)}{2}$
7	$V_{res} \cong (0.6\tau_m A_m + 0.4 N) \leq 0.065 f_m$	$M_{res} = A_s f_{yd} z$ Where $z = \left(l + \frac{b}{2}\right) \left(1 - 0.5 \frac{A_s f_{yd}}{h \left(l + \frac{b}{2}\right) f_d}\right) \leq 0.95 \left(l + \frac{b}{2}\right)$

2) To compute the shear strength in the Mexican seismic code the value of initial shear strength τ_m is multiplied by 0.5. The 100 % of the vertical loads and a friction coefficient of 0.3 are used. The area corresponds in this case to the area of the confined masonry wall (A_t) as defined in Figure 1, including the area of tie-columns.

To compute the flexural capacity strength the same assumptions used in the Argentinean seismic code are used in the Mexican and Chilean codes.

3) In the shear strength formula of the Chilean seismic code the first term of the expression to compute the allowable shear resistance is 0.23 times the shear strength of the masonry and the second term 0.12 times the compressive stress given by the 100 % of the vertical loads. The ultimate shear strength of the confined masonry wall (V_{res}) is obtained after multiplying the allowable strength by 2.5. This last value was obtained in experimental tests developed in Chile.

4) The Colombian seismic code does not use the initial shear strength of the masonry computed by a test setup as the rest of the codes. Instead is computed as the square root of the compressive strength of the masonry. The shear strength depends also on the acting vertical loads, the gross section of the masonry wall and a friction coefficient of 0.33.

The expression used to compute the theoretical flexural capacity is only applied if vertical loads are smaller than $0.10 f_m A_m$. If they are not, no formulation is given by this code to compute the theoretical flexural capacity.

5) As in the Colombian seismic code, the Costa Rican code also used the square root of the compressive masonry strength. A factor of 0.5 is used to take into account the effects of seismic load reversal and the friction coefficient is taken equal to 0.3.

In this code and in the Peruvian seismic code the theoretical flexural capacity is obtained with the expression used for reinforced masonry.

6) In the Peruvian seismic code the initial shear strength is multiplied by a factor of 0.5 or 0.35, depending on the type of masonry bricks (0.5 for clay and concrete blocks, 0.35 for calcium silicate ones). The friction coefficient is assumed to be 0.23. The gross section used corresponds to the area of the confined masonry (including columns).

7) Another denomination is adopted in the case of the Eurocodes due to the fact that confined masonry shear strength is computed using the value of the initial shear strength determined by a different testing procedure. The panel shear strength (V_{res}) varies with the initial shear strength (f_{vo}) and the compressive stress. The wall length to be used in this verification corresponds to the length $l_c = l - b$ (from Figure 1) of the masonry element. A friction coefficient of 0.4 is assumed.

COMPARISON

The different expressions given for the lateral strength of confined masonry are based on the same resisting parameters. The maximum compressive strength of the confined masonry panel, N_u , is approximately the same for all the code expressions. However, there are some differences in the determination of the shear strength of

masonry. In the Colombian and Costa Rican code, this value is assumed to be the square root of the compressive strength of masonry and the Eurocodes use a different strength parameter in comparison with the other Latin America codes.

As it can be observed in Figure 2 all the minimum curves obtained correspond to the curve of shear strength at least to a value of 0.55 the normalized vertical load. Typical values of vertical load ratios are below this value in confined masonry buildings.

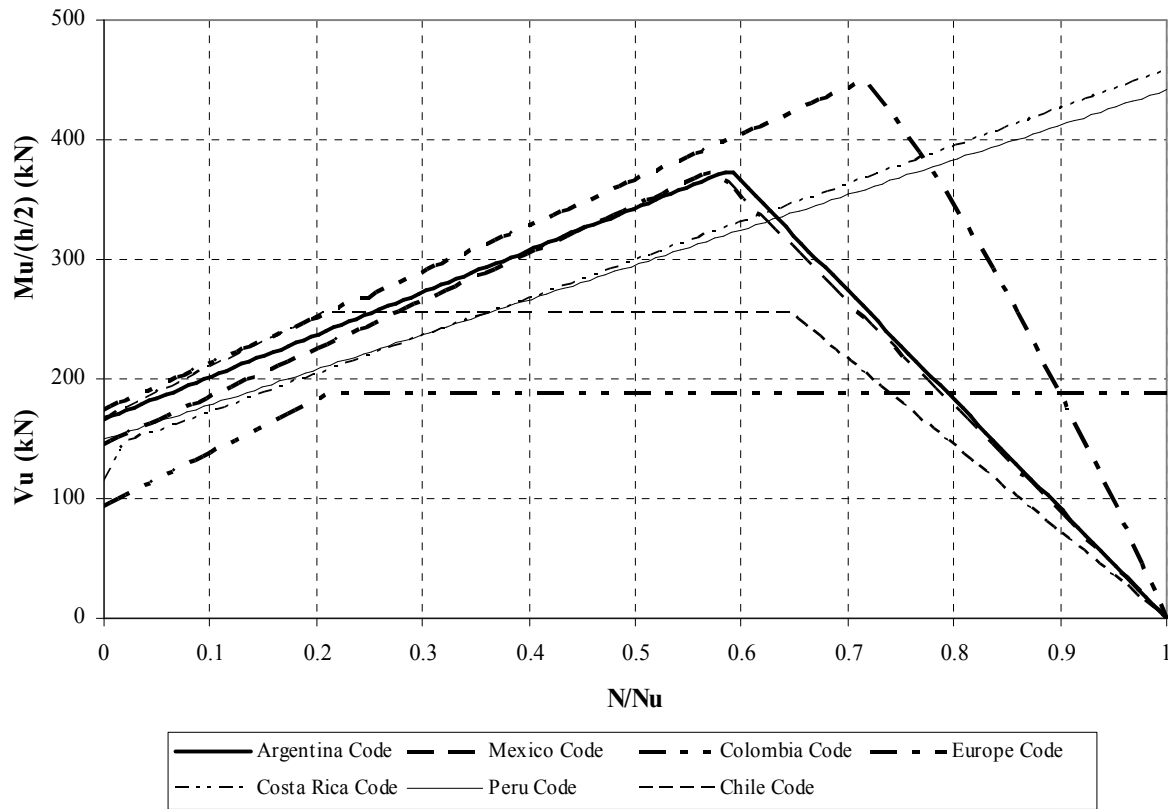


FIGURE 2 Comparison of the minimum curves obtained with different code formulation

In some cases, as the Peruvian and Costa Rican codes, only the shear strength criterion is used. This both curves tend to be very close even if the determination of the initial shear strength is different.

In the Colombian and the Chilean codes the limit to the maximum shear governs in a significant range of normalized vertical loads.

COMPARISON OF EXPERIMENTAL AND THEORETICAL RESULTS

A comparison between eleven experimental results and theoretical values is described next. The theoretical values were obtained using the code expressions for shear and flexural capacity given above. On the other hand the experimental tests used were developed in Chile ¹⁰, Argentina ^{11, 12}, Italy ¹³, Venezuela ¹⁴, Peru ¹⁵ and Mexico ¹⁶. The characteristic of all the specimens tested are summarized in Table 2 and 3. Mean values were used to compute the theoretical strengths.

TABLE 2 Description of the specimens: type, dimensions of the panels and applied vertical loads

Test	Country	Year	Masonry panels	Confining elements	Vertical load (kN)
1	Chile	2004	Concrete bricks masonry walls 3650x2250 - thickness: 140 mm	150x150 mm	0
2	Chile	2004	Hollow clay brick masonry walls 3600x2200 - thickness: 140 mm	150x150 mm	0
3	Argentina San Juan	2004	Solid clay bricks masonry 3000x3000 - thickness: 180 mm	200x200 mm	100
4	Argentina San Juan	2004	Solid clay bricks masonry 3000x3000 - thickness: 180 mm	200x200 mm	200
5	Italy	2006	AAC bricks masonry 1530x2765 - thickness: 300 mm	Ø 200 mm	200
6	Italy	2006	AAC bricks masonry 2900x2765 - thickness: 300 mm	Ø 200 mm	300
7	Venezuela	2004	Hollow concrete blocks 3000x2300 - thickness: 150 mm	150x150 mm	135
8	Peru	2005	Handicraft concrete blocks 2700x2585 - thickness: 200 mm	200x200 mm	0
9	Mexico	1999	Hollow clay brick masonry walls 2500x2500 - thickness: 120 mm	150x150 mm	117
10	Argentina Cordoba	1985	Solid clay bricks masonry 2350x2010 - thickness: 125 mm	150x150 mm	0
11	Argentina Cordoba	1985	Hollow clay brick masonry walls 2350x2010 - thickness: 120 mm	150x150 mm	0

The results of the comparison are shown in Figure 3 were minimum theoretical values given by codes either using shear or flexural capacity are plotted against the experimental ones. Almost all of the theoretical values were lower than the test results. The lowest theoretical results were obtained with the Colombian and the Costa Rican codes making them the most conservative ones. Both of these codes used the square root of the compressive strength to determine the shear strength instead of using a test setup to compute the initial shear strength. This assumption can be a reason of the lowest values obtained. On the other hand the rest of the codes expressions give a good approximation to experimental values.

The majority of the minimum theoretical values are given by the shear expressions. As in correspondence with the conclusions given above most of the codes tend to be conservative and use the shear strength theoretical expressions to verify and/or design confined masonry structures.

TABLE 3 Mechanical parameters of the masonry, reinforcement of tie-columns and maximum obtained resisting values

Test	Longitudinal reinforcement of tie-columns	Masonry compressive strength f_m (MPa)	Masonry shear strength (Mpa)		Yield strength of reinforcement f_y (MPa)	Test average value (kN)
			f_{vo}	τ_m		
1	4ø10	6.04	0.29	0.49	420	123
2	4ø10	6.89	0.33	0.55	420	177
3	4ø10	5.0	0.18	0.3	420	106
4	4ø16	5.0	0.18	0.3	420	221
5	4ø12	2.2	0.24	0.38	430	100
6	4ø12	2.2	0.24	0.38	430	250
7	4ø12.7	6.67	0.30	0.5	412	203
8	4ø12.7	1.65	0.15	0.25	412	109
9	4ø16	11.5	0.65	1.08	410	204
10	4ø8	1.96	0.13	0.22	412	50
11	4ø6	2.45	0.10	0.17	412	31

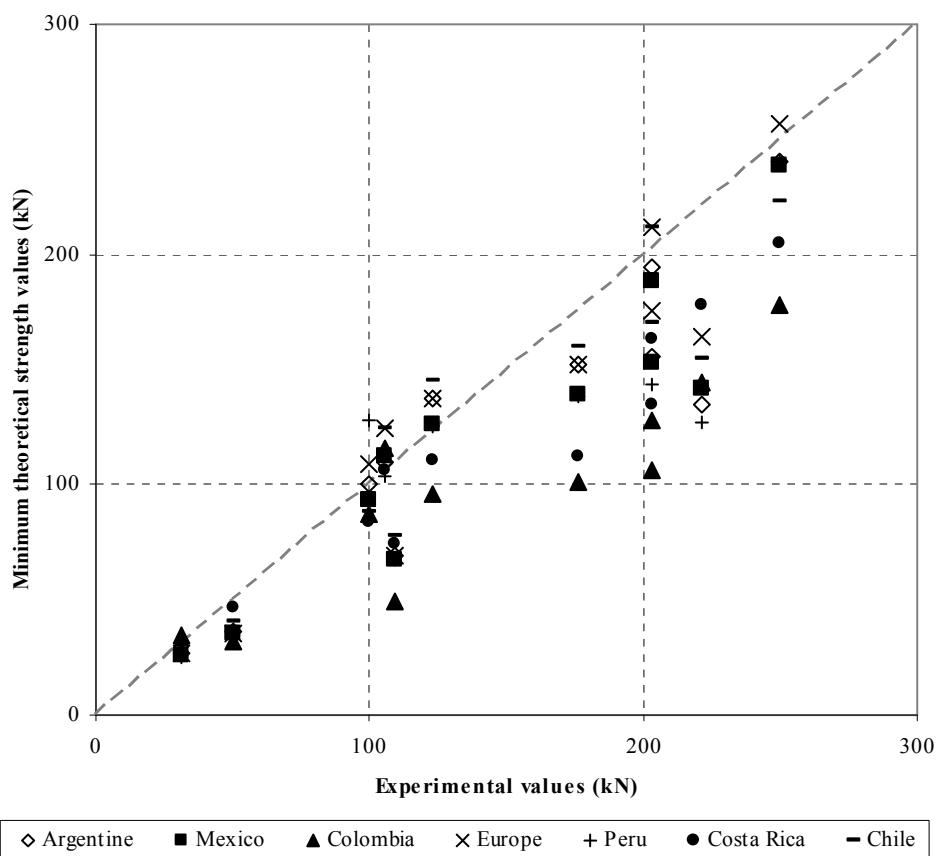


FIGURE 3 Comparison between experimental and minimum values obtained from shear and flexural capacity expressions

In order to clarify the previous results of the comparison between theoretical and experimental values the correlation coefficient was computed. From the graphic, the approximation of the different code expressions to the experimental values can be clearly identified. The Colombian code gives the lowest correlation coefficient and the following lesser corresponds to the Costa Rican seismic code.

On the other hand the Argentinean, Mexican, European and Chilean codes have an approximately equal coefficient of correlation of 0.9.

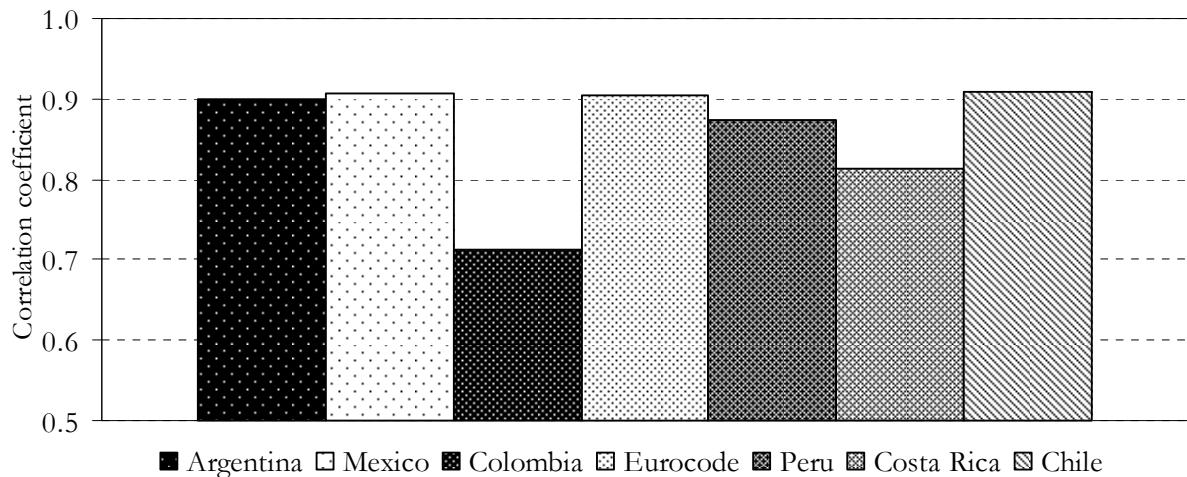


FIGURE 4 Correlation between theoretical and experimental results

CONCLUSIONS

Different code formulations exist nowadays for the verification of the lateral resistance of confined masonry walls. These expressions correspond to shear and flexural theoretical verifications, based in the same mechanical parameters.

The lateral verification generally corresponds to theoretical expressions used to compute the shear strength giving minimum values compared to the flexural theoretical expressions. The comparison with the experimental results confirm the used of shear strength expressions for the lateral verification as it gives the minimum values.

Some of the considers design codes allow under certain hypothesis to neglect the flexural safety checks due to the fact that in most cases in the event of an earthquake the confined masonry wall with usual values of vertical compression and slenderness ratios is expected to fail in shear.

A marked difference in the determination of the initial shear strength is observed being a cause of the lowest values obtained for the Colombian and Costa Rican seismic codes. Similar results were obtained for the Eurocodes and the rest of the Latin America codes (in particular the Argentinean, Mexican and Chilean codes) even if the shear strength parameter is not the same.

REFERENCES

1. Argentinean seismic code: INPRES-CIRSOC 103 [1991] "Normas Argentinas para construcciones sismorresistentes". Parte III. Construcciones de Mampostería.
2. Mexican seismic code: Gaceta Oficial del Distrito Federal [2004] "Normas técnicas complementarias para diseño y construcción de estructuras de mampostería". Tomo I. No. 103-BIS.
3. Chilean seismic code: NCh2123 [1997] "Albañilería confinada: Requisitos de diseño"
4. Colombian seismic code: NSR-98 [1998] "Mampostería estructural". Título D
5. Costa Rican seismic code: CSCR-02 [2002] "Código sísmico de Costa Rica. Capítulo 9: Requisitos para mampostería estructural"
6. Peruvian seismic code: E.070. [2006] "Norma técnica E.070 de albañilería".
7. Eurocode 6 [1996] "Design of masonry structures – Part 1-1: Common rules for reinforced and unreinforced masonry structures"
8. Eurocode 8 [1998] "Design provisions for earthquake resistance of structures. Part 1-2: General rules- General rules for buildings"
9. Magenes G., Calvi G. M., Gaia F. [1996] "Shear tests on reinforced masonry walls". *Serie rapporti scientifici. Rapporto RS-03/96*.
10. Yáñez F., Astroza M., Holmberg A., Ogaz O. [2004] "Behaviour of confined masonry shear walls with large openings". *Proceedings of 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada. Paper no 3438.
11. Decanini L., Payer A., Serrano C., Terzariol R. [1985] "Investigación experimental sobre el comportamiento sismorresistente de prototipos a escala natural de muros de mampostería encadenada". *XXIII Jornadas Sudamericana de Ingeniería estructural*.
12. Zabala F., Bustos J.L., Masanet, A., Santalucía J. [2004] "Experimental behaviour of masonry structural walls used in Argentina". *Proceedings of 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada. Paper no 1093.
13. Penna, A., Magenes, G., Calvi, G.M., and Costa, A.A. [2008] "Seismic performance of AAC infill and bearing walls with different reinforcement solutions", *Proceedings of the 14th International Brick and Block Masonry Conference*, Sydney, Australia.
14. Marinilli A., Castilla E. [2004] "Experimental evaluation of confined masonry walls with several confining-columns" *Proceedings of 13th World Conference on Earthquake Engineering*. Vancouver, B.C., Canada. Paper no 1093.
15. Quiun D., Alferez K., Quinto D. [2005] "N A16-03 Reforzamiento estructural de muros de albañilería de bloques artesanales de concreto" *Congreso Chileno de Sismología e Ingeniería Antisísmica IX Jornadas*, Concepción, Chile.
16. Alcocer, S.M., Zepeda, J.A. [1999] "Behaviour of multi-perforated clay bricks walls under earthquake-type loading," *Proceedings of the 8th North American Masonry Conference*, Austin, TX, USA, June 1999. 235-246.